10.5.2 For composite girders, the ratio of the overall depth of girder (concrete slab plus steel girder) to the length of span preferably should not be less than 0.045 for simple spans and 0.04 for continuous spans.

10.5.3 For trusses the ratio of depth to length of span preferably should not be less than 0.1.

10.5.4 *Deleted*

10.5.5 The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.[*]

10.6 DEFLECTION

10.6.1 The term "deflection" as used herein shall be the deflection computed in accordance with the assumption made for loading when computing the stress in the member.

10.6.2 Members having simple or continuous spans preferably should be designed so that the ratio of the deflection to the length of the span due to service live load plus impact shall not exceed $^{1}/_{800}$, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall not exceed $^{1}/_{1000}$.

10.6.3 The ratio of the deflection to the cantilever arm length due to service live load plus impact preferably should be limited to $^{1}/_{300}$ except for the case including pedestrian use, where the ratio preferably should be $^{1}/_{375}$.

10.6.4 When spans have cross-bracing or diaphragms sufficient in depth or strength to ensure lateral distribution of loads, the deflection may be computed for the standard H or HS loading considering all beams or stringers as acting together and having equal deflection.

10.6.5 The moment of inertia of the gross cross-sectional area shall be used for computing the deflections of beams and girders. When the beam or girder is a part of a composite member, the service live load may be considered as acting upon the composite section.

10.6.6 The gross area of each truss member shall be used in computing deflections of trusses. If perforated plates are used, the effective area shall be the net volume divided by the length from center to center of perforations.

[*] For consideration to be taken into account when exceeding these limitations, reference is made to "Bulletin No.19, Criteria for the Deflection of Steel Bridges," available from the American Iron and Steel Institute, Washington, D.C.

10.6.7 The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.[*]

10.7 LIMITING LENGTHS OF MEMBERS

10.7.1 For compression members, the slenderness ratio, *KL/r*, shall not exceed 120 for main members, or those in which the major stresses result from dead or live load, or both; and shall not exceed 140 for secondary members, or those whose primary purpose is to brace the structure against lateral or longitudinal force, or to brace or reduce the unbraced length of other members, main or secondary.

10.7.2 In determining the radius of gyration, *r*, for the purpose of applying the limitations of the *KL/r* ratio, the area of any portion of a member may be neglected provided that the strength of the member as calculated without using the area thus neglected and the strength of the member as computed for the entire section with the *KL/r* ratio applicable thereto, both equal or exceed the computed total force that the member must sustain.

10.7.3 The radius of gyration and the effective area of a member containing perforated cover plates shall be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates the cross-sectional area of the member shall be considered the same as for a section having perforations in the same transverse plane.

10.7.4 The unbraced length, *L*, shall be assumed as follows:

> For the compression chords of trusses, the length between panel points laterally supported as indicated under Article 10.16.12; for other members, the length between panel point intersections or centers of braced points or centers of end connections.

10.7.5 For tension members, except rods, eyebars, cables, and plates, the ratio of unbraced length to radius of gyration shall not exceed 200 for main members, shall not exceed 240 for bracing members, and shall not exceed 140 for main members subject to a reversal of stress.

10.8 MINIMUM THICKNESS OF METAL

10.8.1 The plate thickness of structural steel including bracing, cross frames, and all types of gusset plates, shall be not less than $^{5}/_{16}$ inch. The web thickness of rolled beams or channels shall be not less than 0.23 inches. The thickness of closed ribs in orthotropic decks, fillers, and in railings, shall be not less than $^{3}/_{16}$ inch.

10.8.2 Where the metal will be exposed to marked corrosive influences, it shall be increased in thickness or specially protected against corrosion.

10.8.3 It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc. As stated above, fillers need not be $^{5}/_{16}$ inch minimum.

10.8.4 For compression members, refer to "Trusses" (Article 10.16).

10.8.5 For flexural members, refer to "Plate Girders" (Article 10.34).

10.8.6 For stiffeners and outstanding legs of angles, etc., refer to relevant Articles 10.10, 10.34, 10.37, 10.48, 10.51 and 10.55.

10.9 EFFECTIVE NET AREA FOR TENSION MEMBERS

10.9.1 When a tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective net area A_e is equal to the net area A_n.

10.9.2 When a tension load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the member, the effective net area A_e shall be calculated as:

$$A_e = UA \tag{10-1a}$$

where:

A = area as defined below (in.^{2})
U = reduction coefficient
= $1 - (x/L)$ 0.9 or as defined in (c) and (d)
x = connection eccentricity (in.); for rolled or built-up shapes, it is referred to the center of gravity of the material lying on either side of the centerline of symmetry of the cross-section, as shown in Fig. 10.9.2A
L = length of connection in the directions of loading (in.)

Larger values of U are permitted to be used when justified by tests or other rational criteria.

(a) When the tension load is transmitted only by bolts or rivets:

$A = A_n$ = net area of member (in.^{2})

(b) When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds:

$A = A_g$ = gross area of member (in.^{2})

(c) When the tension load is transmitted only by transverse welds:

A = area of directly connected elements (in.^{2})
U = 1.0

(d) When the tension load is transmitted to a plate by longitudinal welds along both edges at the end of the plate for $L_w > W$

A = area of plate (in.^{2})

for L_w $2W$	U = 1.0
for $2W > L_w$ $1.5\,W$	U = 0.87
for $1.5W > L_w$ W	U = 0.75

where:

L_w = length of weld (in.)
W = plate width (distance between welds) (in.)

10.9.3 *Deleted*

10.9.4 *Deleted*

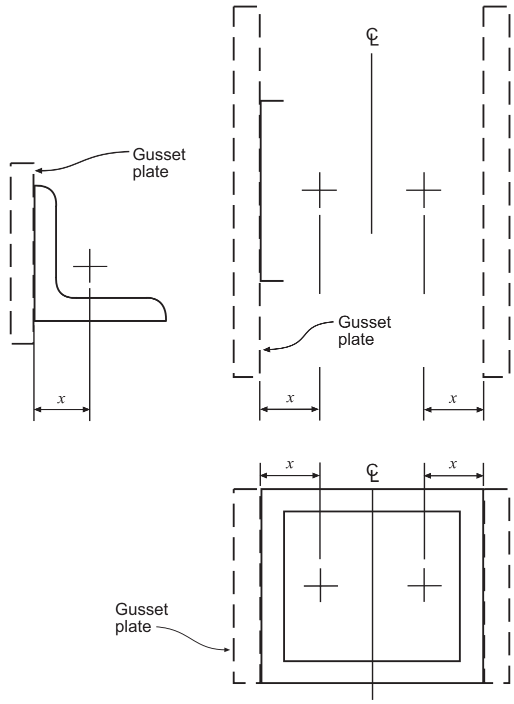

FIGURE 10.9.2A Determination of *x*

10.10 OUTSTANDING LEGS OF ANGLES

The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following:

In main members carrying axial compression load, 12 times the thickness.

In bracing and other secondary members, 16 times the thickness.

For other limitations see Article 10.35.2.

10.11 EXPANSION AND CONTRACTION

In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provisions shall be made for changes in length of span resulting from live loads. In spans more than 300 feet long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement.

10.12 MEMBERS

10.12.1 Flexural Members

Flexural members shall be designed using the elastic section modulus except when utilizing compact sections under Strength Design as specified in Articles 10.48.1, 10.50.1.1, and 10.50.2.1. In determining flexural strength, the gross section shall be used, except that if more than 15 percent of each flange area is removed, that amount removed in excess of 15 percent shall be deducted from the gross area. In no case shall the design tensile stress on the net section exceed 0.50 F_u, when using service load design method or 1.0 F_u, when using strength design method, where F_u equals the specified minimum tensile strength of the steel, except that for M 270 Grades 100/100W steels the design tensile stress on the net section shall not exceed 0.46 F_u when using the service load design method.

10.12.2 Compression Members

The strength of compression members connected by high-strength bolts and rivets shall be determined by the gross section.

10.12.3 Tension Members

The strength of tension members connected by bolts or rivets shall be determined by the gross section unless the net section area is less than 85 percent of the corresponding gross area, in which case that amount removed in excess of 15 percent shall be deducted from the gross area. In no case shall the design tensile stress on the net section exceed 0.50 F_u, when using service load design method or 1.0 F_u, when using strength design method, where F_u equals the specified minimum tensile strength of the steel, except that for M 270 Grades 100/100W steels the design tensile stress on the net section shall not exceed 0.46 F_u when using the service load design method.

10.13 COVER PLATES

10.13.1 The length of any cover plate added to a rolled beam shall be not less than (2*d* + 36) in. where (*d*) is the depth of the beam (in.).

10.13.2 Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the member.

10.13.3 The maximum thickness of a single cover plate on a flange shall not be greater than 2 times the thickness of the flange to which the cover plate is attached. The total thickness of all cover plates should not be greater than $2^1/_2$ times the flange thickness.

10.13.4 Any partial length welded cover plate shall extend beyond the theoretical end by the terminal distance, and it shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal adjacent to or connected by fillet welds. The theoretical end of the cover plate, when using service load design methods, is the section at which the stress in the flange without that cover plate equals the allowable service load stress, exclusive of fatigue considerations. When using strength design methods, the theoretical end of the cover plate is the section at which the flange strength without that cover plate equals the required strength for the design loads, exclusive of fatigue requirements. The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends, and $1^1/_2$ times for cover plates welded across their ends. The width at ends of tapered cover plates shall be not less that 3 inches. The weld connecting the cover plate to the flange in its terminal distance shall be continuous and of sufficient size to develop a total force of not less than the computed force in the cover plate at its theoretical end. All welds connecting cover plates to beam flanges shall be continuous and shall not be smaller than the minimum size permitted by Article 10.23.2.2.

10.13.5 Any partial length end-bolted cover plate shall extend beyond the theoretical end by the terminal distance equal to the length of the end-bolted portion, and the cover plate shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal at ends of partial length welded cover plates with high-strength bolted, slip-critical end connections (Table 10.3.1B). Beams with end-bolted cover plates shall be fabricated in the following sequence: drill holes; clean faying surfaces; install bolts; weld. The theoretical end of the end-bolted cover plate is determined in the same manner as that of a welded cover plate, as specified in Article 10.3.4. The bolts in the slip-critical connections of the cover plate ends to the flange, shall be of sufficient numbers to develop a total force of not less than the computed force in the cover plate at the theoretical end. The slip resistance of the end-bolted connection shall be determined in accordance with Article 10.32.3.2 for service load design, and 10.56.1.4 for load factor design. The longitudinal welds connecting the cover plate to the beam flange shall be continuous and stop a distance equal to one bolt spacing before the first row of bolts in the end-bolted portion.

10.14 CAMBER

Girder should be cambered to compensate for dead load deflections and vertical curvature required by profile grade.

10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

10.15.1 Scope

This section pertains to rolled beans and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified minimum yield strength greater than 50,000 psi, except for Grade HPS 70W Steel, shall not be heat-curved. +

10.15.2 Minimum Radius of Curvature

10.15.2.1 For heat-curved beams and girders, the horizontal radius of curvature measured to the centerline of the girder web shall not be less than 150 feet and shall not be less than the larger of the values calculated (at any and all cross sections throughout the length of the girder) from the following two equations:

$$R = \frac{440\, b\, D}{\sqrt{F_y y}\; t_w} \qquad (10\text{-}1)$$ +

$$R = \frac{7{,}500{,}000\, b}{F_y y} \qquad (10\text{-}2)$$ +

where:

F_y = specified minimum yield strength of the web (psi) +

y = ratio of the total cross-sectional area to the cross-sectional area of both flanges
b = widest flange width (in.)
D = clear distance between flanges (in.)
t_w = web thickness (in.)
R = horizontal radius of curvature (in.)

10.15.2.2 In addition to the above requirements, the radius shall not be less than 1,000 feet when the flange thickness exceeds 3 inches or the flange width exceeds 30 inches.

10.15.3 Camber

To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, the amount of camber, Δ (in.) at any section along the length L of the girder shall be equal to:

$$\Delta = \frac{\Delta_{DL}}{\Delta_M}\left(\Delta_M + \Delta_R\right) \qquad (10\text{-}3)$$

$$\Delta_R = \frac{0.02 L^2 F_y}{E Y_o}\left(\frac{1{,}000 - R}{850}\right) \geq 0$$

where:

Δ_{DL} = camber at any point along the length L calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads (in.)
Δ_M = maximum value of Δ_{DL} within the length L (in.)
E = modulus of elasticity of steel (psi)
F_y = specified minimum yield strength of girder flange (psi)
Y_o = distance from the neutral axis to the extreme outer fiber (in.) (maximum distance for non-symmetrical sections)
R = radius of curvature (ft.)
L = span length for simple spans or for continuous spans, the distance between a simple end support and the dead load contraflexure point, or the distance between points of dead load contraflexure (in.)

Camber loss between dead load contraflexure points adjacent to piers is small and may be neglected.

Note: Part of the camber loss is attributable to construction loads and will occur during construction of the bridge; total camber loss will be complete after several months of in-service loads. Therefore, a portion of the camber increase (approximately 50 percent) should be included in the bridge profile. Camber losses of this nature (but generally smaller in magnitude) are also known to occur in straight beams and girders.

10.16 TRUSSES

10.16.1 General

10.16.1.1 Component parts of individual truss members may be connected by welds, rivets, or high-strength bolts.

10.16.1.2 Preference should be given to trusses with single intersection web systems. Members shall be symmetrical about the central plane of the truss.

10.16.1.3 Trusses preferably shall have inclined end posts. Laterally unsupported hip joints shall be avoided.

10.16.1.4 Main trusses shall be spaced a sufficient distance apart, center to center, to be secure against overturning by the design lateral forces.

10.16.1.5 For the calculation of forces, effective depths shall be assumed as follows:

Riveted and bolted trusses, distance between centers of gravity of the chords.

Pin-connected trusses, distance between centers of chord pins

10.16.2 Truss Members

10.16.2.1 Chord and web truss members shall usually be made in the following shapes:

"H" sections, made with two side segments (composed of angles or plates) with solid web, perforated web, or web of stay plates and lacing.

Channel sections, made with two angle segments, with solid web, perforated web, or web of stay plates and lacing.

Single Box sections, made with side channels, beams, angles, and plates or side segments of plates only, connected top and bottom with perforated plates or stay plates and lacing.

Single Box sections, made with side channels, beams, angles and plates only, connected at top with solid cover plates and at the bottom with perforated plates or stay plates and lacing.

Double Box sections, made with side channels, beams, angles and plates or side segments of plates only, connected with a conventional solid web, together with top and bottom perforated cover plates or stay plates and lacing.

10.16.2.2 If the shape of the truss permits, compression chords shall be continuous.

10.16.2.3 In chords composed of angles in channel shaped members, the vertical legs of the angles preferably shall extend downward.

10.16.2.4 If web members are subject to reversal of stress, their end connections shall not be pinned. Counters preferably shall be rigid. Adjustable counters, if used, shall have open turnbuckles, and in the design of these members an allowance of 10,000 psi shall be made for initial stress. Only one set of diagonals in any panel shall be adjustable. Sleeve nuts and loop bars shall not be used.

10.16.3 Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Secondary stresses due to truss distortion or floor beam deflection usually need not be considered in any member, the width of which, measured parallel to the plane of distortion, is less than one-tenth of its length. If the secondary stress exceeds 4,000 psi for tension members and 3,000 psi for compression members, the excess shall be treated as a primary stress. Stresses due to the flexural dead load moment of the member shall be considered as additional secondary stress.

10.16.4 Diaphragms

10.16.4.1 There shall be diaphragms in the trusses at the end connections of floor beams.

10.16.4.2 The gusset plates engaging the pedestal pin at the end of the truss shall be connected by a diaphragm. Similarly, the webs of the pedestal shall, if practicable, be connected by a diaphragm.

10.16.4.3 There shall be a diaphragm between gusset plates engaging main members if the end tie plate is 4 feet or more from the point of intersection of the members.

10.16.5 Camber

The length of the truss members shall be such that the camber will be equal to or greater than the deflection produced by the dead load.

10.16.6 Working Lines and Gravity Axes

10.16.6.1 Main members shall be proportioned so that their gravity axes will be as nearly as practicable in the center of the section.

10.16.6.2 In compression members of unsymmetrical section, such as chord sections formed of side segments and a cover plate, the gravity axis of the section shall coincide as nearly as practicable with the working line, except that eccentricity may be introduced to counteract dead load bending. In two-angle bottom chord or diagonal members, the working line may be taken as the gage line nearest the back of the angle or at the center of gravity for welded trusses.

10.16.7 Portal and Sway Bracing

10.16.7.1 Through truss spans shall have portal bracing, preferably, of the two-plane or box type, rigidly connected to the end post and the top chord flanges, and as deep as the clearance will allow. If a single plane portal is used, it shall be located, preferably, in the central transverse plane of the end posts, with diaphragms between the webs of the posts to provide for a distribution of the portal stresses. The portal bracing shall be designed to take the full end reaction of the top chord lateral system, and the end posts shall be designed to transfer this reaction to the truss bearings.

10.16.7.2 Through truss spans shall have sway bracing 5 feet or more deep at each intermediate panel point. Top lateral struts shall be at least as deep as the top chord.

10.16.7.3 Deck truss spans shall have sway bracing in the plane of the end posts and at all intermediate panel points. This bracing shall extend the full depth of the trusses below the floor system. The end sway bracing shall be proportioned to carry the entire upper lateral load to the supports through the end posts of the truss.

10.16.8 Perforated Cover Plates

When perforated cover plates are used, the following provisions shall govern their design.

10.16.8.1 The ratio of length, in direction of stress, to width of perforation, shall not exceed two.

10.16.8.2 The clear distance between perforations in the direction of stress shall not be less than the distance between points of support.

10.16.8.3 The clear distance between the end perforation and the end of the cover plate shall not be less than 1.25 times the distance between points of support.

10.16.8.4 The point of support shall be the inner line of fasteners or fillet welds connecting the perforated plate to the flanges. For plates butt welded to the flange edge of rolled segments, the point of support may be taken as the weld whenever the ratio of the outstanding flange width to flange thickness of the rolled segment is less than seven. Otherwise, the point of support shall be the root of the flange of the rolled segment.

10.16.8.5 The periphery of the perforation at all points shall have a minimum radius of $1\frac{1}{2}$ inches.

10.16.8.6 For thickness of metal, see Article 10.35.2.

10.16.9 Stay Plates

10.16.9.1 Where the open sides of compression members are not connected by perforated plates, such members shall be provided with lacing bars and shall have stay plates as near each end as practicable. Stay plates shall be provided at intermediate points where the lacing is interrupted. In main members, the length of the end stay plates between end fasteners shall be not less than $1\frac{1}{4}$ times the distance between points of support and the length of intermediate stay plates not less than $\frac{3}{4}$ of that distance. In lateral struts and other secondary members, the overall length of end and intermediate stay plates shall be not less than $\frac{3}{4}$ of the distance between points of support.

10.16.9.2 The point of support shall be the inner line of fasteners or fillet welds connecting the stay plates to the flanges. For stay plates butt welded to the flange edge of rolled segment, the point of support may be taken as the weld whenever the ratio of outstanding flange width to flange thickness or the rolled segment is less than seven. Otherwise, the point of support shall be the root of flange of rolled segment. When stay plates are butt welded to rolled segments of a member, the allowable stress in the member shall be determined in accordance with Article 10.3. Terminations of butt welds shall be ground smooth.

10.16.9.3 The separate segments of tension members composed of shapes may be connected by perforated plates or by stay plates or end stay plates and lacing. End stay plates shall have the same minimum length as specified for end stay plates on main compression members, and intermediate stay plates shall have a minimum length of $\frac{3}{4}$ of that specified for intermediate stay plates on main compression members. The clear distance between stay plates on tension members shall not exceed 3 feet.

10.16.9.4 The thickness of stay plates shall be not less than $\frac{1}{50}$ of the distance between points of support for main members, and $\frac{1}{60}$ of that distance for bracing members. Stay plates shall be connected by not less than three fasteners on each side, and in members having lacing bars the last fastener in the stay plates preferably shall also pass through the end of the adjacent bar.

10.16.10 Lacing Bars

When lacing bars are used, the following provisions shall govern their design.

10.16.10.1 Lacing bars of compression members shall be so spaced that the slenderness ratio of the portion of the flange included between the lacing bar connections will be not more than 40 or more than $\frac{2}{3}$ of the slenderness ratio of the member.

10.16.10.2 The section of the lacing bars shall be determined by the formula for axial compression in which L is taken as the distance along the bar between its connections to the main segments for single lacing, and as 70 percent of that distance for double lacing.

10.16.10.3 If the distance across the member between fastener lines in the flanges is more than 15 inches and a bar with a single fastener in the connection is used, the lacing shall be double and fastened at the intersections.

10.16.10.4 The angle between the lacing bars and the axis of the member shall be approximately 45 degrees for double lacing and 60 degrees for single lacing.

10.16.10.5 Lacing bars may be shapes or flat bars. For main members, the minimum thickness of flat bars shall be $^1/_{40}$ of the distance along the bar between its connections for single lacing and $^1/_{60}$ for double lacing. For bracing members, the limits shall be $^1/_{50}$ for single lacing and $^1/_{75}$ for double lacing.

10.16.10.6 The diameter of fasteners in lacing bars shall not exceed one-third the width of the bar. There shall be at least two fasteners in each end of lacing bars connected to flanges more than 5 inches in width.

10.16.11 Gusset Plates

10.16.11.1 Gusset or connection plates preferably shall be used for connecting main members, except when the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member shall be given consideration. The gusset plates shall be designed to resist shear, axial force, and bending moments acting on the weakest or critical section.

10.16.11.2 Re-entrant cuts, except curves made for appearance, shall be avoided as far as practicable.

10.16.11.3 If the length of unsupported edge of a gusset plate exceeds the value of the expression $11{,}000/\sqrt{F_y}$ times its thickness, the edge shall be stiffened.

10.16.11.4 Listed below are the values of the expression $11{,}000/\sqrt{F_y}$ for the following grades of steel:

F_y (psi)	$11{,}000/\sqrt{F_y}$
36,000	58
50,000	49
70,000	42
90,000	37
100,000	35

10.16.12 Half-Through Truss Spans

10.16.12.1 The vertical truss members and the floor beams and their connections in half-through truss spans shall be proportioned to resist a lateral force of not less than 300 pounds per linear foot applied at the top chord panel points of each truss.

10.16.12.2 The compression chord shall be designed as a compression member with elastic lateral supports at the panel points. The strength of the compression chord, so determined, shall exceed the maximum force from dead load, live load, and impact in any panel of the compression chord by not less than 50 percent.*

10.16.13 Fastener Pitch in Ends of Compression Members

In the ends of compression members, the pitch of fasteners connecting the component parts of the member shall not exceed four times the diameter of the fastener for a length equal to $1^1/_2$ times the maximum width of the member. Beyond this point, the pitch shall be increased gradually for a length equal to $1^1/_2$ times the maximum width of the member until the maximum pitch is reached.

10.16.14 Net Section of Riveted or High-Strength Bolted Tension Members

10.16.14.1 The net section of a riveted or high-strength bolted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width.

* For a discussion of columns with elastic lateral supports, refer to Timoshenko & Gere, "Theory of Elastic Stability," McGraw-Hill Book Co., Second Edition, P.70.

10.16.14.2 The net width for any chain of holes extending progressively across the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding, for each gage space in the chain, the quantity:

$$\frac{s^2}{4g} \tag{10-4}$$

where:

s = pitch of any two successive holes in the chain (in.)
g = gage of the same holes (in.)

The net section of the part is obtained from the chain that gives the least net width.

10.16.14.3 For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of gages from back of angle less the thickness.

10.16.14.4 At a splice, the total force in the member being spliced is transferred by fasteners to the splice material.

10.16.14.5 When determining the stress on any least net width of either splice material or member being spliced, the amount of the force previously transferred by fasteners adjacent to the section being investigated shall be considered in determining the stress on the net section.

10.16.14.6 The diameter of the hole shall be taken as $^1/_8$ inch greater than the nominal diameter of the rivet or high-strength bolt, unless larger holes are permitted in accordance with Article 10.24.

10.17 BENTS AND TOWERS

10.17.1 General

Bents preferably shall be composed of two supporting columns, and the bents usually shall be united in pairs to form towers. The design of members for bents and towers is governed by applicable articles.

10.17.2 Single Bents

Single bents shall have hinged ends or else shall be designed to resist bending.

10.17.3 Batter

Bents preferably shall have a sufficient spread at the base to prevent uplift under the design lateral loadings. In general, the width of a bent at its base shall be not less than one-third of its height.

10.17.4 Bracing

10.17.4.1 Towers shall be braced, both transversely and longitudinally, with stiff members having either welded, high-strength bolted or riveted connections. The sections of members of longitudinal bracing in each panel shall not be less than those of the members in corresponding panels of the transverse bracing.

10.17.4.2 The bracing of long columns shall be designed to fix the column about both axes at or near the same point.

10.17.4.3 Horizontal diagonal bracing shall be placed in all towers having more than two vertical panels, at alternate intermediate panel points.

10.17.5 Bottom Struts

The bottom struts of towers shall be strong enough to slide the movable shoes with the structure unloaded, the coefficient of friction being assumed at 0.25. Provision for expansion of the tower bracing shall be made in the column bearings.

10.18 SPLICES

10.18.1 General

10.18.1.1 Design Strength

Splices may be made by rivets, by high-strength bolts, or by the use of welding. In general, splices whether in tension, compression, bending, or shear, shall be designed in the cases of the service load design or the strength design methods for a capacity based on not less than 100 percent of the allowable design strength in the member taking into account the bolt holes. Bolted and riveted splices in flexural members shall satisfy the requirements of Article 10.18.2. Bolted and riveted splices in compression members shall satisfy the requirements of Article 10.18.3. Bolted and riveted splices in tension members shall satisfy the requirements of Article 10.18.4. Bolted and riveted splices in tension members shall also satisfy the requirements of Article 10.19.4. Welded splices shall satisfy the requirements of Article 10.18.5. Where a section changes at a splice, the small section is to be used to satisfy the above splice requirements.

10.18.1.2 Fillers

10.18.1.2.1 For fillers 1/4 inch and thicker in bolted or riveted axially loaded connections, including girder flange splices, additional fasteners shall be required to distribute the total stress in the member uniformly over the combined section of the member and the filler. The filler shall either be extended beyond the splice material and secured by additional fasteners, or as an alternate to extending the filler, an equivalent number of fasteners may be included in the connection. Fillers 1/4 inch and thicker need not be extended and developed provided that the design shear strength of the fasteners, specified in Article 10.56.1.3.2 in the case of the strength design method and in the Tables 10.32.3A and 10.32.3B in the case of the service load design method, is reduced by the following factor R:

$$R=[(1+\gamma)/(1+2\gamma)] \qquad (10\text{-}4a)$$

where $\gamma = A_f / A_p$

A_f = sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

The design slip force, specified in Article 10.56.1.3.2 in the case of the strength design method and in Article 10.32.3.2.1 in the case of the service load design method, for slip-critical connections shall not be adjusted for the effect of the fillers. Fillers 1/4 inch or more in thickness shall consist of not more than two plates, unless special permission is given by the Engineer.

10.18.1.2.2 For bolted web splices with thickness differences of 1/16 inch or less, no filler plates are required.

10.18.1.2.3 Fillers for welded splices shall conform to the requirements of the *AISI/AASHTO/AWS D1.5 Bridge Welding Code.*

10.18.1.3 Design Force for Flange Splice Plates

For a flange splice with inner and outer splice plates, the flange design force may be assumed to be divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than 10 percent. When the areas of the inner and outer plates differ by more than 10 percent, the design force in each splice plate and its connection shall be determined by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the shear strength of the connection shall be checked for the maximum calculated splice plate force acting on a single shear plane. The slip resistance of high-strength bolted connections for a flange splice with inner and outer splice plates shall always be checked for the flange design force divided equally to the two slip planes.

10.18.1.4 Truss Chords and Columns

Splices in truss chords and columns shall be located as near to the panel points as practicable and usually on the side where smaller stress occurs. The arrangement of plates, angles, or other splice elements shall be such as to make proper provision for the stresses, both axial and bending, in the component parts of the member spliced.

10.18.2 Flexural Member

10.18.2.1 General

10.18.2.1.1 Splices shall preferably be made at or near points of dead load contraflexure in continuous spans and at points of the section change.

10.18.2.1.2 In both flange and web splices, there shall be not less than two rows of bolts on each side of the joint.

10.18.2.1.3 Oversize or slotted holes shall not be used in either the member or the splice plates at the bolted splices.

10.18.2.1.4 In both flange and web splices, high-strength bolted connections shall be proportioned to prevent slip during erection of the steel and during the casting or placing of the deck.

10.18.2.1.5 Deleted

10.18.2.1.6 Flange and web splices in areas of stress reversal shall be checked for both positive and negative flexure.

10.18.2.1.7 Riveted and bolted flange angle splices shall include two angles, one on each side of the flexural member.

10.18.2.2 Flange Splices

10.18.2.2.1 For checking the strength of flange splices, an effective area, A_e, shall be used for the flanges and for the individual splice plates as follows:

For flanges and their splice plates subject to tension:

$$A_e = w_n t + \beta A_g \leq A_g \qquad (10\text{-}4b)$$

where:

W_n = least net width of the flange or splice plate computed as specified in Article10.16.14 (in.)
t = flange or splice plate thickness (in.)
A_g = gross area of the flange or splice plate (in.^{2})
β = 0.0 for M 270 Grade 100/100W steels, or when holes exceed 1 1/4 inch in diameter
= 0.15 for all other steels and when holes are less than or equal to 1 1/4 inch in diameter

The diameter of the holes shall be taken as specified in Article 10.16.14.6.

For the flanges and their splice plates subject to compression:

$$A_e = A_g \qquad (10\text{-}4c)$$

10.18.2.2.2 In the case of the strength design method, the splice plates shall be proportioned for a design force, P_{cu} equal to a design stress, F_{cu}, times the smaller effective area, A_e, on either side of the splice. F_{cu} is defined as follows:

$$F_{cu} = \alpha F_{yf} \qquad (10\text{-}4d)$$

where:

α = 1.0 except that a lower value equal to (M_u/M_y) may be used for flanges in compression at sections where M_u is less than M_y

M_u = design bending strength of the section in positive or negative flexure at the point of splice, whichever causes the maximum compressive stress due to the factored loads at the mid-thickness of the flange under consideration (lb-in.)
M_y = moment capacity at first yield for the section at the point of splice used to compute M_u (lb-in.). For composite sections, M_y shall be calculated in accordance with Article10.50(c). For hybrid sections, M_y shall be computed in accordance with Article 10.53.
F_{yf} = specified minimum yield strength of the flange (psi)

In calculating M_u and M_y, holes in the flange subject to tension shall be accounted for as specified in Article 10.12. For a flange splice with inner and outer splice plates, the flange design forces shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, A_e, of each splice plate shall be sufficient to prevent yielding of the splice plate under its calculated portion of the design force. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied.

As a minimum, high-strength bolted connection for both top and bottom flange splices shall be proportioned to prevent slip at an overload design force, P_{fo}, defined as follows:

$$P_{fo} = \left| f_o / R \right| A_g \qquad (10\text{-}4e)$$

where:

f_o = maximum flexural stress due to Group I loading divided by 1.3 at the mid-thickness of the flange under consideration for the smaller section at the point of splice (psi)
R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when f_o is less than or equal to the specified minimum yield strength of the web, F_{yw}. For homogeneous girders, R shall always be taken equal to 1.0

A_g = smaller gross flange area on either side of the splice (in.^{2})

f_o and R shall be computed using the gross section of the member. The slip resistance of the connection shall be computed from Equation (10-172).

10.18.2.2.3 In the case of the service load design method, the splice plates shall be proportioned for a design force, P_{cf} equal to the allowable flexural stress for the flange under consideration at the point of splice, F_b, times the smaller effective area, A_e, on either side of the splice.

For a flange splice with inner and outer splice plates, the flange design forces shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, A_e, of each splice plate shall be sufficient to ensure that the stress in the splice plate does not exceed the allowable flexural stress under its calculated portion of the design force. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Table 10.3.23B. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied.

As a minimum, high-strength bolted connection for both top and bottom flange splices shall be proportioned to prevent slip at a force equal to the flange design stress times the smaller value of the gross flange area on either side of the splice. The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1.

10.18.2.2.4 (Deleted)

10.18.2.3 Web Splices

10.18.2.3.1 In general, web splice plates and their connections shall be proportioned for shear, a moment due to eccentricity of the shear at the point of splice, and a portion of the flexural moment that is assumed to be resisted by the web at the point of splice. Webs shall be spliced symmetrically by plates on each side. The web splice plates shall extend as near as practical for the full depth between flanges.

10.18.2.3.2 In the case of the strength design method, web splice plates and their connections shall be proportioned for a design shear, V_{wu} equal to the shear capacity of the smaller web at the point of splice, V_u.

10.18.2.3.3 In the case of the strength design method, web splice plates and their connections shall be proportioned for a design moment, M_{vu} due to the eccentricity of the design shear at the point of splice defined as follows:

$$M_{vu} = V_{wu} e \tag{10-4f}$$

where:

V_{wu} = design shear in the web at the point of splice defined in Article 10.18.2.3.2 (lb.)

e = distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration (in.)

10.18.2.3.4 In the case of the strength design method, web splice plates and their connections shall be proportioned for a design moment, M_{wu}, representing the portion of the flexural moment that is assumed to be resisted by the web. M_{wu} shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H_{wu}, shall also be applied at the mid-depth of the web. M_{wu} and H_{wu} may be computed as follows:

For non-compact sections:

$$M_{wu} = \frac{t_w D^2}{12}\left(RF_{cr} + F_{yf}\right) \tag{10-4g}$$

$$H_{wu} = \frac{t_w D}{2}\left(F_{yf} - RF_{cr}\right) \tag{10-4h}$$

For compact sections:

$$M_{wu} = \frac{t_w F_{yw}}{4}\left(D^2 - 4y_c^2\right) \tag{10-4i}$$

$$H_{wu} = 2 t_w y_o F_{yw} \tag{10-4j}$$

where:

F_{cr} = design flexural strength specified in Articles 10.50.1.2 and 10.50.2.2 for composite sections, or determined by $M_u/S_{xc}R_b$, where M_u is defined as in Articles 10.48.2, 10.48.3, 10.48.4 for noncomposite sections (psi)

F_{yf} = specified minimum yield strength of the flange (psi)

F_{yw} = specified minimum yield strength of the web (psi)

y_o = distance form the mid-depth of the web to the plastic neutral axis (in.)

D = clear unsupported distance between flange components (in.)

t_w = web thickness (in.)

10.18.2.3.5 In the case of the strength design method, web splice plates and their connections shall be proportioned for the most critical combination of V_{wu}, M_{vu}, M_{wu}, and H_{wu}. The connections shall be proportioned as eccentrically loaded connections to resist the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of: 1) an overload design shear, V_{wo}, 2) an overload design moment, M_{vo}, due to the eccentricity of the overload design shear, 3) an overload design moment, M_{wo}, applied at mid-depth of the web representing the portion of the flexural moment that is assumed to be resisted by the web, and 4) for sections where the neutral axis is not located at mid-depth of the web, an overload horizontal design force H_{wo}, applied at mid-depth of the web, as follows:

$$V_{wo} = V_o \qquad (10\text{-}4k)$$

$$M_{vo} = V_{wo}e \qquad (10\text{-}4l)$$

where:

V_o = maximum shear in the web due to Group I loading divided by 1.3 at the point of splice (lb.)

M_{wo} and H_{wo} may be determined as follows:

$$M_{wo} = \frac{t_w D^2}{12}\left| f_o - f_{of} \right| \qquad (10\text{-}4m)$$

$$H_{wo} = \frac{t_w D}{2}\left(f_o + f_{of} \right) \qquad (10\text{-}4n)$$

where:

f_o = maximum flexural stress due to Group I loading divided by 1.3 at the mid-thickness of the flange under consideration for smaller section at the point of splice (positive for tension; negative for compression) (psi)

f_{of} = flexural stress due to Group I loading divided by 1.3 at the mid-thickness of the other flange at the point of splice concurrent with f_o in the flange under consideration (positive for tension; negative for compression) (psi)

f_o and f_{of} shall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Equation (10-172) with N_b taken equal to 1.0.

10.18.2.3.6 In the case of the service load design method, web splice plates and their connections shall be proportioned for a design shear stress in the web at the point of splice, F_w equal to the allowable shear stress in the web at the point of splice, F_v.

10.18.2.3.7 In the case of the service load design method, web splice plates and their connections shall be proportioned for a design moment, M_v due to the eccentricity of the design shear at the point of splice defined as follows:

$$M_v = F_w D t_w e \qquad (10\text{-}4o)$$

where:

F_w = design shear stress in the web at the point of splice defined in Article 10.18.2.3.6 (psi)

D = web depth (in.)

t_w = web thickness (in.)

10.18.2.3.8 In the case of the service design method, web splice plates and their connections shall be proportioned for a design moment, M_w, representing the portion of the flexural moment that is assumed to be resisted by the web. M_w shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H_w, shall also be applied at the mid-depth of the web. M_w and H_w may be computed as follows:

$$M_w = \frac{t_w D^2}{12}\left(RF_{bc} + F_{bt} \right) \qquad (10\text{-}4p)$$

$$H_w = \frac{t_w D}{2}\left(F_{bt} - RF_{bc}\right) \qquad (10\text{-}4q)$$

where:

F_{bc} = allowable compression flange stress specified in Table 10.32.1A (psi)

F_{bt} = allowable tension flange stress specified in Table 10.32.1A (psi)

10.18.2.3.9 In the case of the service load design method, web splice plates and their connections shall be proportioned for the most critical combination of F_wDt_w, M_v, M_w, and H_w. The connections shall be proportioned as eccentrically loaded connections to resist the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Table 10.32.3B. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of F_wDt_w, M_v, M_w, and H_w. M_w, and H_w shall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Table 10.32.3.2.1 with N_b taken equal to 1.0.

10.18.3 Compression Members

Compression members such as columns and chords shall have ends in close contact at riveted and bolted splices. Splices of such members which will be fabricated and erected with close inspection and detailed with milled ends in full contact bearing at the splices may be held in place by means of splice plates and rivets or high-strength bolts proportioned for not less than 50 percent of the lower allowable design strength of sections spliced. The strength of compression members connected by high-strength bolts or rivets shall be determined using the gross section.

10.18.4 Tension Members

The tension strength of splice components shall be based on Article 10.12.3. For calculating the net section, the provisions of Articles 10.9 and 10.16.14 shall apply.

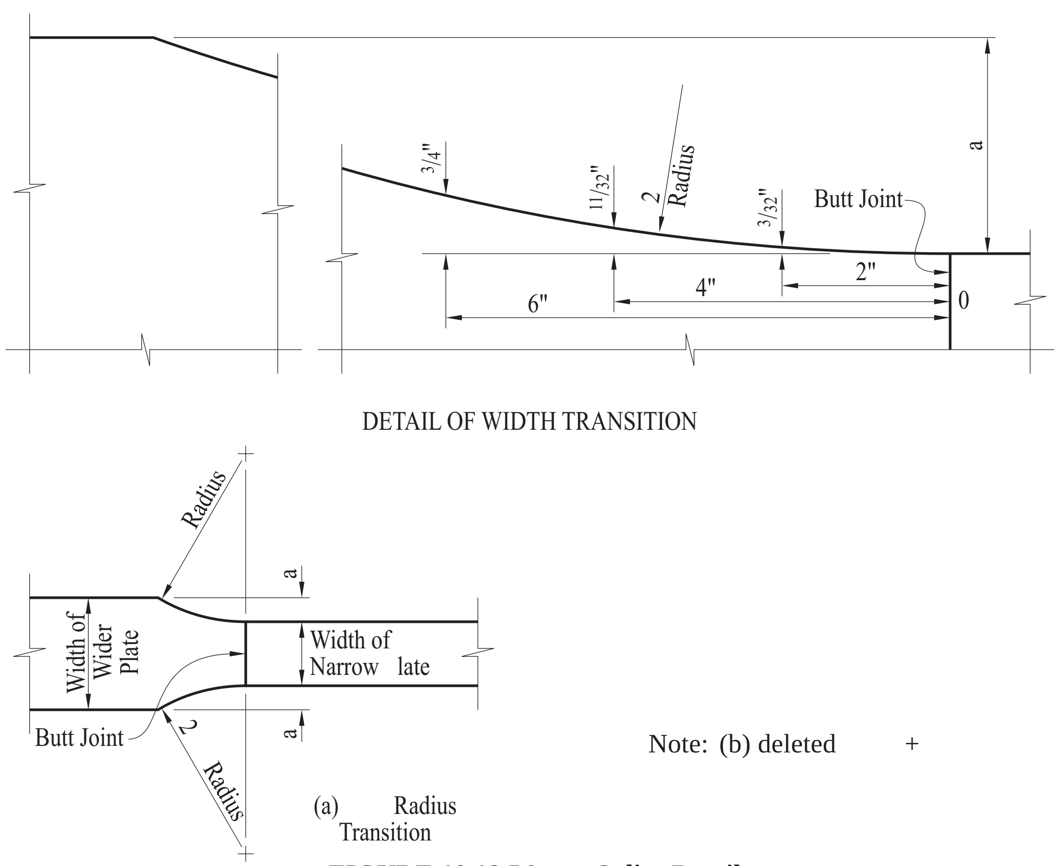

FIGURE 10.18.5A Splice Details

As a minimum, in the case of the strength design method, high-strength bolted connections for splices in tension members shall be proportioned to prevent slip at an overload design force, P_o, equal to the maximum tensile stress in the member due to Group I loading divided by 1.3 times the gross section of member. The slip resistance shall be computed from Equation (10-172). In the case of the service load design method, high-strength bolted connections shall be proportioned to prevent slip at a force equal to the allowable design strength specified in Article 10.18.1.1 times the gross area of the member. The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1.

10.18.5 Welding Splices

10.18.5.1 Tension and compression members may be spliced by means of full penetration butt welds, preferably without the use of splice plates.

10.18.5.2 Splices shall not be welded in field.

10.18.5.3 Material of different widths spliced by butt welds shall have transitions conforming to Figure 10.18.5A. At butt weld splices joining material of different thicknesses there shall be a uniform slope between the offset surfaces of not more than 1 in 2 ${}^{1}/_{2}$ with respect to the surface of either part.

10.19 CONNECTIONS

10.19.1 General

10.19.1.1 Except as otherwise provided herein, connections for main members shall be designed in the cases of the service load design and the strength design methods for a capacity based on not less than 100 percent of the allowable design strength in the member.

10.19.1.2 Connections shall be made symmetrical about the axis of the members insofar as practicable. Connections, except for lacing bars and handrails, shall contain not less than two fasteners or equivalent weld.

10.19.1.3 Members, including bracing, preferably shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided, if practicable, but if unavoidable the members shall be so proportioned that the combined forces will not exceed the allowable design strength.

10.19.1.4 In the case of connections which transfer total member shear at the end of the member, the gross section shall be taken as the gross section of the connected elements.

10.19.2 End Connections of Floor Beams and Stringers

10.19.2.1 The end connection shall be designed for calculated member loads. The end connection angles of floor beams and stringers shall be not less than ${}^{3}/_{8}$ inch in finished thickness. Except in cases of special end floor beam details, each end connection for floor beams and stringers shall be made with two angles. The length of these angles shall be as great as the flanges will permit. Bracket or shelf angles which may be used to furnish support during erection shall not be considered in determining the number of fasteners required to transmit end shear.

10.19.2.2 End connection details shall be designed with special care to provide clearance for making the field connection.

10.19.2.3 End connections of stringers and floor beams preferably shall be bolted with high-strength bolts; however, they may be riveted or welded. In the case of welded end connections, they shall be designed for the vertical loads and the end bending moment resulting from the deflection of the members.

10.19.2.4 Where timber stringers frame into steel floor beams, shelf angles with stiffeners shall be provided to carry the total reaction. Shelf angles shall be not less than ${}^{7}/_{16}$ inch thick.

10.19.3 End Connections of Diaphragms and Cross Frames

10.19.3.1 The end connections for diaphragms or cross frames in straight rolled-beam and plate girder bridges shall be designed for the calculated member loads.

10.19.3.2 Vertical connection plates such as transverse stiffeners which connect diaphragms or cross frames to the beam or girder shall be rigidly connected to both top and bottom flanges.

10.19.4 Block Shear Rupture Strength

10.19.4.1 General

Block shear rupture is one of several possible failure modes for splices, connections, gusset plates and tension members. Block shear rupture failure is developed when the net section of one segment ruptures and the gross section of a perpendicular segment yields. The web connections of coped beams, all tension connections including connection plates, splice plates and gusset plates, and tension members shall be investigated to ensure that the adequate block shear rupture strength is provided.

10.19.4.2 Allowable Block Shear Rupture Stress

In the Service Load Design Method, calculated tension stress based on the gross section shall not exceed the allowable block shear rupture stress obtained from the following equations:

for $A_{tn} \geq 0.6\,A_{vn}$

$$F_{bs} = \left(0.33\,F_y A_{vg} + 0.55 F_u A_{tn}\right) / \,A_g \qquad \text{(10-4r)}$$

for $A_{tn} < 0.6 A_{vn}$

$$F_{bs} = \left(0.33\,F_u A_{vn} + 0.55\,F_y A_{tg}\right) / \,A_g \qquad \text{(10-4s)}$$

where:

A_g = gross area of whole connected material (in.^{2})
A_{vg} = gross area along the plane resisting shear (in.^{2})
A_{vn} = net area along the plane resisting shear (in.^{2})
A_{tg} = gross area along the plane resisting tension (in.^{2})
A_{tn} = net area along the plane resisting tension (in.^{2})
F_y = specified minimum yield strength of the connected materials (psi)
F_u = specified minimum tensile strength of the connected materials (psi)
F_{bs} = allowable block shear rupture stress (psi)

10.19.4.3 Design Block Shear Rupture Strength

In the Strength Design Method, calculated tension force shall not exceed the design block shear rupture strength obtained from the following equations:

for $A_{tn} \geq 0.58\,A_{vn}$

$$T_{bs} = \phi_{bs}\left(0.58\,F_y A_{vg} + F_u A_{tn}\right) \qquad \text{(10-4t)}$$

for $A_{tn} < 0.58\,A_{vn}$

$$T_{bs} = \phi_{bs}\left(0.58\,F_u A_{vn} + F_y A_{tg}\right) \qquad \text{(10-4u)}$$

where:
T_{bs} = design block shear rupture strength (lb.)
ϕ_{bs} = 0.8, reduction factor for block shear rupture strength

10.20 DIAPHRAGMS AND CROSS FRAMES

10.20.1 General

Rolled beam and plate girder spans shall be provided with cross frames or diaphragms at each support and with cross frames or diaphragms placed in all bays and spaced at intervals not to exceed 25 feet. Diaphragms for rolled beams shall be at least $^1/_3$ and preferably $^1/_2$ the beam depth and for plate girders shall be at least $^1/_2$ and preferably $^3/_4$ the girder depth. Cross frames shall be as deep as practicable. Cross frames shall preferably be of the cross type or vee type. End cross frames or diaphragms shall be proportioned to adequately transmit all the lateral forces to the bearings. Intermediate cross frames shall be normal to the main beams and girders when the supports are skewed more than twenty degrees (20°). Cross frames on horizontally curved steel girder bridges shall be designed as main members with adequate provisions for transfer of lateral forces from the girder flanges. Cross frames and diaphragms shall be designed for horizontal wind loads as described in Article 10.21.2, seismic loads and other applicable loads.

10.20.2 Horizontal Force

The maximum horizontal force, F_D (lb.), in the transverse diaphragms and cross frames is obtained from the following:

$$F_D = 1.14WS_d \tag{10-5}$$

where:
W = wind load along the exterior flange (lb/ft)
S_d = diaphragm spacing (ft)

10.20.2.1 *Deleted*

10.20.2.2 *Deleted*

10.20.3 *Deleted*

10.21 LATERAL BRACING

10.21.1 The need for lateral bracing shall be investigated for wind loads, seismic loads and other applicable lateral loads. Flanges attached to concrete decks or other decks of comparable rigidity will not require lateral bracing.

10.21.2 A horizontal wind force of 50 pounds per square foot shall be applied to the area of the superstructure exposed in elevation. Half this force shall be applied in the plane of each flange. The maximum induced stresses, f (psi), in the bottom flange of each girder in the system when top flanges are continuously supported can be computed from the following:

$$f = R\,f_{cb} \tag{10-6}$$

$$R = \left[0.2272L - 11\right]S_d^{-2/3} \quad \left(\begin{array}{l}\text{when no bottom lateral}\\ \text{bracing is provided}\end{array}\right) \tag{10-7}$$

$$R = \left[0.059L - 0.64\right]S_d^{-1/2} \quad \left(\begin{array}{l}\text{when bottom lateral}\\ \text{bracing is provided}\end{array}\right) \tag{10-8}$$

$$f_{cb} = \frac{72\,M_{cb}}{t_f b_f^2} \tag{10-9}$$

$$M_{cb} = 0.08W\,S_d^2 \tag{10-10}$$

L = span length (ft.)
t_f = thickness of flange (in.)
b_f = width of flange (in.)

The stresses in flanges of each girder in the system when top flanges are not continuously supported shall be computed using the structural system in the plane of the flanges under consideration. The allowable stress shall be factored in accordance with Article 3.22.

10.21.3 When required, lateral bracing shall be placed in the exterior bays between diaphragms or cross frames. All required lateral bracing shall be placed in or near the plane of the flange being braced.

10.21.4 Where beams or girders comprise the main members of through spans, such members shall be stiffened against lateral deformation by means of gusset plates or knee braces with solid webs which shall be connected to the stiffeners on the main members and the floor beams. If the unsupported length of the edge of the gusset plate (or solid web) exceeds 60 times its thickness, the plate or web shall have a stiffening plate or angles connected along its unsupported edge.

10.21.5 Through truss spans, deck truss spans, and spandrel braced arches shall have top and bottom lateral bracing.

10.21.6 Bracing shall be composed of angles, other shapes, or welded sections. The smallest angle used in bracing shall be 3 by $2^1/_2$ inches. There shall be not less than 2 fasteners or equivalent weld in each end connection of the angles.

10.21.7 If a double system of bracing is used, both systems may be considered effective simultaneously if the members meet the requirements both as tension and compression members. The members shall be connected at their intersections.

10.21.8 The lateral bracing of compression chords preferably shall be as deep as the chords and effectively connected to both flanges.

10.22 CLOSED SECTIONS AND POCKETS

10.22.1 Closed sections, and pockets or depressions that will retain water, shall be avoided where practicable.

Pockets shall be provided with effective drain holes or be filled with waterproofing material.

10.22.2 Details shall be so arranged that the destructive effects of bird life and the retention of dirt, leaves, and other foreign matter will be reduced to a minimum. Where angles are used, either singly or in pairs, they preferably shall be placed with the vertical legs extending downward. Structural tees preferably shall have the web extending downward.

10.23 WELDING

10.23.1 General

10.23.1.1 Steel base metal to be welded, weld metal, and welding design details shall conform to the requirements of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code* and the current Standard Specifications of the California Department of Transportation.

10.23.1.2 Welding symbols shall conform with the latest edition of the American Welding Society Publication AWS A2.4.

10.23.1.3 Fabrication shall conform to the Standard Specifications of the California Department of Transportation. For fracture critical members see the *AASHTO "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members."*

10.23.2 Effective Size of Fillet Welds

10.23.2.1 Maximum Size of Fillet Welds

The maximum size of a fillet weld that may be assumed in the design of a connection shall be such that the stresses in the adjacent base material do not exceed the values allowed in Article 10.32. The maximum size that may be used along edges of connected parts shall be:

(1) Along edges of material less than $^1/_4$ inch thick, the maximum size may be equal to the thickness of the material.

(2) Along edges of material $^1/_4$ inch or more in thickness, the maximum size shall be $^1/_{16}$ inch less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

10.23.2.2 Minimum Size of Fillet Welds

The minimum fillet weld size shall be as shown in the following table**.

Base Metal Thickness of Thicker Part Jointed (T)		Minimum Size of Fillet Weld*		
in.	mm	in.	mm	
$T \leq {}^3/_4$	T 19.0	$^1/_4$	6	Single-pass Welds must be used
$T > {}^3/_4$	T> 19.0	$^5/_{16}$	8	

* Except that the weld size need not exceed the thickness of the thinner part jointed. For this exception, particular care should be taken to provide sufficient preheat to ensure weld soundness.

** Smaller fillet welds may be approved by the Engineer based upon applied stress and the use of appropriate preheat.

10.23.3 Minimum Effective Length of Fillet Welds

The minimum effective length of a fillet weld shall be four times its size and in no case less than $1^1/_2$ inches.

10.23.4 Fillet Weld End Returns

Fillet welds which support a tensile force that is not parallel to the axis of the weld, or which are proportioned to withstand repeated stress, shall not terminate at corners of parts or members but shall be returned continuously, full size, around the corner for a length equal to twice the weld size where such return can be made in the same plane. End returns shall be indicated on design and detail drawings.

10.23.5 Seal Welds

Seal welding shall preferably be accomplished by a continuous weld combining the functions of sealing and strength, changing section only as the required strength or the requirements of minimum size fillet weld, based on material thickness, may necessitate.

10.24 FASTENERS

10.24.1 General

10.24.1.1 In proportioning fasteners, for shear and tension the cross-sectional area based upon the nominal diameter shall be used. Galvanization of AASHTO M253 (ASTM A490) and A354 Grade BD high strength bolts is not permitted due to hydrogen embrittlement problems. These fasteners must be carefully evaluated before being utilized. Requirements for bolts in these specifications shall be used for threaded rods, threaded studs and anchor rods, where applicable.

10.24.1.2 High-strength bolts may be substituted for Grade 1 rivets (ASTM A 502) or ASTM A307 bolts. When AASHTO M 164 (ASTM A325) high-strength bolts are substituted for ASTM A307 bolts they shall be tightened to the full effort of a man using an ordinary spud wrench.

10.24.1.3 All bolts, except high-strength bolts tensioned to the requirements of the Standard Specifications of the California Department of Transportation, shall have single self-locking nut, double nuts, or a nut with a thread locking system (anaerobic adhesive) to prevent nut loosening. The thread locking system is the preferred method for bolt diameters of one inch or less. The thread locking system shall not be used on bolt diameters greater than one inch. When using the double nut method a torque value for the jam nut, relative to the main nut, shall be shown on the plans to assure that a reasonable effort will be made to lock the two nuts together.

10.24.1.4 Joints required to resist shear between their connected parts are designated as either slip-critical or bearing-type connections. Slip-critical joints are required for joints subject to stress reversal, heavy impact loads, severe vibration or where stress and strain due to joint slippage would be detrimental to the serviceability of the structure. They include:

(1) Joints subject to fatigue loading.
(2) Joints with bolts installed in oversized holes.
(3) Except where the Engineer intends otherwise and so indicates in the contract documents, joints with bolts installed in slotted holes where the force on the joint is in a direction other than normal (between approximately 80 and 100 degrees) to the axis of the slot.
(4) Joints subject to significant load reversal.
(5) Joints in which welds and bolts share in transmitting load at a common faying surface.
(6) Joints in which, in the judgment of the Engineer, any slip would be critical to the performance of the joint or the structure and so designated on the contract plans and specifications.

10.24.1.5 High-strength bolted connections subject to tension, or combined shear and tension shall be designed as slip-critical connections.

10.24.1.6 Bolted bearing-type connections using high-strength bolts shall be limited to members in compression and secondary members.

10.24.1.7 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than $^3/_8$ inch thick, countersunk fasteners shall not be assumed to carry stress. In metal $^3/_8$ inch thick and over, one-half the depth of countersink shall be omitted in calculating the bearing area.

10.24.1.8 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread run out.

10.24.1.9 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote h or Article 10.56.1.3.).

TABLE 10.24.2 Nominal Hole Dimension

Bolt Diameter (in.)	Hole Dimension (in.)			
	Standard (Diameter)	Oversize (Diameter)	Short Slot (Width × Length)	Long Slot (Width × Length)
$^5/_8$	$^{11}/_{16}$	$^{13}/_{16}$	$^{11}/_{16} \times {}^7/_8$	$^{11}/_{16} \times 1^9/_{16}$
$^3/_4$	$^{13}/_{16}$	$^{15}/_{16}$	$^{13}/_{16} \times 1$	$^{13}/_{16} \times 1^7/_8$
$^7/_8$	$^{15}/_{16}$	$1^1/_{16}$	$^{15}/_{16} \times 1^1/_8$	$^{15}/_{16} \times 2^3/_{16}$
1	$1^1/_{16}$	$1^1/_4$	$1^1/_{16} \times 1^5/_{16}$	$1^1/_{16} \times 2^1/_2$
$1^1/_8$	$d + {}^1/_{16}$	$d + {}^5/_{16}$	$(d + {}^1/_{16}) \times (d + {}^3/_8)$	$(d + {}^1/_{16}) \times (2.5 \times d)$

10.24.2 Hole Types

Hole types for high-strength bolted connections are standard holes, oversize holes, short slotted holes and long slotted holes. The nominal dimensions for each type hole shall be not greater than those shown in Table 10.24.2.

10.24.2.1 In the absence of approval by the Engineer for use of other hole types, standard holes shall be used in high-strength bolted connections.

10.24.2.2 When approved by the Engineer, oversize, short slotted hole or long slotted holes may be used subject to the following joint detail requirements.

10.24.2.2.1 Oversize holes may be used in all plies of connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3, as applicable. Oversize holes shall not be used in bearing-type connections.

10.24.2.2.2 Short slotted holes may be used in any or all plies of high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100 degrees) to the axis of the slot. Short slotted holes may be used without regard for the direction of applied load in any or all plies of connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable.

10.24.2.2.3 Long slotted holes may be used in one of the connected parts at any individual faying surface in high-strength bolted connections designed on the basis of Table 10.32.3B or Table 10.56A, as applicable, provided the load is applied approximately normal (between 80 and 100 degrees) to the axis of the slot. Long slotted holes may be used in one of the connected parts at any individual faying surface without regard for the direction of applied load on connections which satisfy the requirements of Article 10.32.3.2.1 or Article 10.57.3.1, as applicable.

10.24.3 Washer Requirements

Design details shall provide for washers in high-strength bolted connections as follows:

10.24.3.1 Where the outer face of the bolted parts has slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism. Beveled washers other than the standard 1:6 slope shall be detailed in the plans.

10.24.3.2 Hardened washers are not required for connections using AASHTO M164 (ASTM A325) and AASHTO M253 (ASTM A490) bolts except as required in Articles 10.24.3.3 through 10.24.3.7.

10.24.3.3 Hardened washers shall be used under the element turned in tightening and to cover oversize or short slotted holes in the outer ply.

10.24.3.4 Irrespective of the tightening method, hardened washers shall be used under both the head and the nut when AASHTO M253 (ASTM A490) bolts are to be installed in material having a specified yield strength less than 40,000 psi.